CURRENT PROGRESS IN BRIDGE ENGINEERING:

A GERMAN VIEWPOINT

by

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SUBMITTED TO THE DEPARTMENT OF
HUMANITIES IN PARTIAL
FULFILLMENT OF THE
REQUIREMENTS FOR THE
DEGREE OF

BACHELOR OF HUMANITIES AND ENGINEERING

at the

MASSACHUSETTS INSTITUTE OF TECHNOLOGY

June 1982

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Submitted to the Department of Humanities
on May 10, 1982 in partial fulfillment
of the requirements for the Degree of Bachelor of
Science in Humanities and Engineering

ABSTRACT

Two lectures from a seminar on bridge engineering
were translated from the German; two more were rewritten
from notes in English. The lectures discuss the current
status of German bridge engineering, describe newly-
developed construction methods, and detail maintenance
procedures for existing bridges.

The German engineers acknowledged their country's
leading role in bridge technology, which resulted from
the complete rebuilding of Germany following World War
II. In the future, considerations of economics, prefabrication and mechanization will enter into more cons-
struction projects. Earthquake tolerance is an area for
additional research. Civil engineers should have greater
breadth of knowledge and consider the bridge as a part
of the total environment.

Of the new designs used in Germany, cable-stayed bridges and orthotropic steel deck plates are the most
innovative. In a cable-stayed bridge, the cables transmit loads directly from the deck to the tower: in a sus-
pension bridge, in contrast, a catenary with several
secondary vertical cables are used for support. With
bridges of medium spans (100-500m), cable-stays seem
to be the most economical.

An orthotropic steel deck plate integrates the
cross-girders, ribs, tensioning bed, and concrete plate
into a system that can transmit both longitudinal and
transverse loads to the girders. The calculations for
the girder grillages and the composite cross sections are based on the ultimate load bearing capacity of the bridge. Some cable-stayed bridges have used orthotropic plates.

If a design error does not lead to damage in a bridge, then melting salts and pervious concrete covers and joints might lead to maintenance problems. Even difficulties that arise during construction can cause later harm to a structure. Thus, it is necessary to inspect bridges before, during and after construction. Simpler, less-sensitive structures also experience fewer difficulties.
PREFACE

When completed, this work will be sent to West-Berlin for final proofreading prior to its publication as a booklet later in the year. The booklet will be a selling point for two groups at the Massachusetts Institute of Technology and the Technical University of Berlin, who hope to set up a joint research/experimental program on bridge engineering.

On Sept. 14-17, Professors J. Lindner (speaking in English) and M. Specht (speaking in German) from the TUB held a seminar at M.I.T. on the current state of bridge technology. As a writing student who was fluent in German and who had taken several civil engineering courses, I was asked to do the translations and rewrites. I am proud to contribute my energies and talents to this international research effort.

Because these were originally given orally, there are some inconsistencies and discontinuities that I have not been able to rectify. Where the logic appears faulty, it is because the original lecture notes contained them. I worked from figure headings, which are listed at the back of each chapter.

At M.I.T., Dr. William Bennett, Head of the Writing Department served as my thesis tutor. His green comments and notations helped me correct many stylistic and gram-
matical errors. Professor Oral Buyukozturk, of the Department of Civil Engineering acted as liaison between Berlin and Cambridge, and offered me valuable technical assistance. Jay Rosselini, Assistant Professor of German in the Department of Humanities, made meticulous painstaking corrections to my translations.


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CHAPTER 1

PRESENT STATE OF CONCRETE BRIDGE DESIGN IN GERMANY;
FUTURE DEVELOPMENT OF RESEARCH TOPICS

Prof.-Dr. Engineer Manfred Specht
Technical University of Berlin

In the Federal Republic of Germany today there exist approximately 17 million square meters of bridge surface (Standfuß 1979), more than 80% of which comprise long-span bridges. Roughly two-thirds of these bridges are less than 30 years old and illustrate the phenomenal development of large bridge construction, which has been dominated by the concrete construction (Fig. 1). The consequent new possibilities—the intense reconstruction in an industrial country destroyed by war and densely populated, together with rapid proliferation of automobiles, the stiff competition of a free market economy, and the creative driving force of engineers—led to exceptional achievements.

Of course, this spirited development has become visible mainly in large bridge construction, because there the large-scale projects justify the expense of modern equipment and methods. For today's short-span and medium-span bridges ranging up to about 100m in length, highly rationalized methods for otherwise conventional
techniques are available, especially for building the scaffolding and the formwork, and also for the manufacturing, transporting and placing of the concrete. These bridges, including the pre-cast ones, still contribute more than half the total bridge area.

CLASSIC CANTILEVER CONSTRUCTION

The modern long-span bridge was born in Germany in 1950, when over a tributary of the Rhine, the first reinforced-concrete bridge was erected by the cantilever method: the Balduinstein metal bridge (Fig. 2). At 62.09m, this solid-deck bridge was cast in 3-m units starting simultaneously at both river banks and progressing toward the middle of the river. In retrospect, it served as a "dress rehearsal" at the right moment. Economic considerations were still secondary, and technically it was a complete success. Even today it renders reliable service.

The erection in 1952, of the large river bridge at Worms demonstrated not only the technical perfection but also the economy of cantilever construction. There began the unprecedented success of this construction process developed by U. Finsterwalder (Fig. 3). Up to now, almost 1000 bridges have been constructed world-wide according to this principle. The largest in Germany is the bridge over the Rhine at Bendorf, with a span of 208m and a finished height of 10.40m at the pylons (Fig. 4).
Larger cross-sections are hardly feasible for successful concrete bridges. The practical limit of the span therefore lies around 250m, even with the most effective utilization of material in the cross-section. The Japanese have approached this mark already with spans of 240m. Fig. 5 shows an example: the Urado Bridge, spanning 230m.

Characteristics of classical cantilever construction are:

- The pylons are monolithically jointed to the superstructure (Fig. 6).
- The bridge girder is a two-sided, symmetrical, free cantilever. The concrete sections range from 3 to 5m in length. The construction progresses at a rate of about 1m per day per form carrier (Fig. 7).
- During construction, the system is statically equivalent to the final system. At the outset, a cross-force joint couples both halves at midspan, and has only a moderate reciprocal influence. The redistribution of the sectional forces as a result of creep and shrinkage fall relatively little.
- The moment resulting from the subsequently-added sections of the bridge and from the traffic is carried entirely by
the auxiliary construction equipment (form carrier), which in general was satisfactory. Further construction proceeds without additional reinforcement.

- The longitudinal girders are shaped to resemble a moment diagram, so as to avoid all unnecessary weight.

The middle joint proves to be the weak point of the classical cantilever bridge. Therefore, the last cantilever section completes a monolithic joint between the two halves of the superstructure, thus avoiding formation of an additional joint. Bridges without joints have been built at lengths of 1000m or more. Such structures first became possible after the development of heavy bearings, which could reliably, lastingly and economically endure large displacements due to temperature fluctuations, plastic deformations and resulting settling of the foundation, while simultaneously bearing enormous vertical loads. During construction these bearings must be made flexurally rigid by means of auxiliary structures.

The cantilever construction method made it possible to proceed with the massive beam structures required for larger clear spans. But they could be created economically only if the constituents of the concrete were high-quality and dead weight could be eliminated almost completely.
Therefore, work began early on optimizing the longitudinal girder. A great advantage was the possibility of adjusting the tensioning load vertically on the moment curve that was to be covered, because at the end of every section were anchoring zones, as seen in Fig. 8a. The most appropriate cross-section was the torsionally-rigid box beam.

The cantilever girder that is most practical to build and has the most sensible structure is one with constant strength, that is, with a constant shear force along the length of the bridge axis. (Fig. 9). For a given general loading diagram, there is a definite second-order solution of the inner forces for the progress of the lever arm: a differential equation that can be approximated by a quadratic parabola. A box girder built according to this principle is exceptionally easy to reinforce. From the constant shear force follows the linearity of the slope of the bending tension force or, what is equivalent, the constant increase of the bending moment for each unit of length.

Longitudinal span members arranged as tensile reinforcement are distributed over the entire highway deck and are placed curved toward the web and anchored at equal distances and in equal numbers (Fig. 8b). At their anchorage points these forces must be countered by diagonally-aligned tensile and compressive struts of a covered framework in the web. Spacing of the bracing forces is also equidistant.
Until recently, tensile struts were as a rule constructed in the form of prestressed thrust needles; this approach was the only way to attain the highest value of the shear stress under calculated failure load, according to the standards then current in Germany. That led to over-dimensioning of the tensile struts and to an insufficient estimate of the safety of the compressive braces.

With the new German standard DIN 4227 (1979), this situation improved. Nevertheless, thrust needles still render good service, because the German standards still are interpreted very conservatively, as the graph in Fig. 10 shows. But strong, slack main reinforcement of stirrups always should be part of the primary construction. Unfortunately, a shear theory for the failure mode, which would help eliminate the remaining differences of opinion has yet to be developed.

After the construction of the first large river bridges, an attempt was made to develop further the cantilever method for parallel-flanged bridges of every sort, such as valley bridges, foreland bridges, elevated highways, and so forth. A disadvantage of multiple-span bridges was that the carrier always had to be disassembled, transported (to the other bank of the river, for example), and reassembled on the next pier deck.
Thus a method was needed by which the form carrier could reach the next pier without disassembly. Five different methods solve this difficulty and have proved themselves in practice:

1. Cantilever construction with auxiliary piers,
2. Cantilever construction with auxiliary pylons,
3. Cantilever construction with launching trusses,
4. Cantilever construction with transportation trusses, and
5. Cantilever construction with placing equipment for segments.

CANTILEVER CONSTRUCTION ON AUXILIARY PIERS

An example of this method is the completed freeway bridge over the Mangfall Valley in Bavaria: A framework deck bridge of prestressed concrete with spans of 90m, 108m, 90m. It was completed in 1959 and is an interesting aesthetic solution, which unfortunately will not be economically feasible to duplicate (Fig. 11).

A single carrier travels in one line from abutment to abutment. Whenever the negative moments of the superstructure are exhausted during construction, the superstructure is supported by auxiliary pylons. The largest free cantilever spanned 54m. The auxiliary pylons - in this case concrete ones - were blasted away upon completion of construction (Fig. 12).
CANTILEVER CONSTRUCTION WITH AUXILIARY PYLONS

An auxiliary pylon with cable guys eliminates dependence upon the topography of the site. The pylon may consist of steel or concrete. For the cables, reinforcing steel with a coat of corrosion-resistant paint is used. As guide cables, two basic types have evolved:

(1) The advantage of a fan-shaped cable arrangement (Figs. 13 and 14) is that the cable anchorages can be arranged so that they are concentrated at the top of the pylon and the cables near the pylon are angled very steeply. A disadvantage of this arrangement is the continuously variable curve of the individual cables, a situation requiring special care during installation of the body of the anchorage in the cantilever section.

(2) The anchorage points of a parallel (harp) cable arrangement (Figs. 15 and 16) can be placed at a constant slope by means of a drilling plan. On the other hand, the anchorage zones on the pylon constitute a rather large portion of the height of the pylon. Statically, the harp arrangement is somewhat less efficient.

If the engineer decides to add thrust needles that are essentially parallel, for instance, at an angle less than 45°, then he usually uses the harp configuration. With increasing costs, the immobility
of the pylon stood as an economic obstacle to further development. The mobile launching trusses were the new discovery.

CANTILEVER CONSTRUCTION WITH LAUNCHING TRUSSES

The launching truss, a temporary steel apparatus (Fig. 17), stands midway over the pier and generally above the superstructure. The carrier is somewhat longer than one bridge span. During symmetrical cantilever construction, the truss assumes the weight of the form carrier and the load from the concrete. Section lengths of 5 to 10m have proved to be economical. The already prestressed superstructure portion supports its own weight, although it requires, because of this, somewhat more reinforcing steel than the more recent, statically more favorable continuous system. This steel usually remains in the structure, thereby increasing its safety. During the casting of the concrete, the launching truss assumes the role of a transport bridge for men and material. If a double cantilever arm is constructed, it serves as a traveling track for moving the form carrier. The method proceeds thus:
- the form carrier returns to the initial pylon and is placed on the superstructure,
- the launching truss proceeds to the next pylon,
- the carrier is retrieved, and
- the next double cantilever arm is built.

A less costly variation is the so-called MSF apparatus, in which the launching girder does not remain supported by the top of the pylon, but as construction progresses is placed on the completed portion of the superstructure. Shortening the distance between supports with the additional influence of the weight of the concrete considerably eases the load on the superstructure and makes it less expensive. Spans up to approx. 120m can be constructed economically with this method.

Figs. 18 and 19 show the completed Eiserfeld Bridge over the Sieg Valley in Germany with spans of 105m, concrete sections of 10m and a finished height of 5.40m. The rate of construction was 20m per week. This was a very impressive demonstration of a bridge constructed (in 1969) over settled land using the erection technology that does not remain stationary on the ground.

A final cantilevered segment produces the flexurally rigid final joint. After the concrete hardens, the members
comprising the span can be joined and post-tensioned. Their anchorages lie in the pilaster strips on the deck plate or on the upper side of the web. By means of their increasing angles of inclination (slopes), they reduce the transverse load. Fig. 20 shows a construction detail from the plans; Fig. 21 shows the completed structure. The excellent camouflaging of the working joints is easily recognized. But the anchorage zones on the upper side of the web remain a problem, at least as long as there is no real additional deck caulking to guard against the chlorine-rich, dewy salt water. Even with systems constructed of the highest quality materials, points of failure appear. Yet the anchorage zone must be protected before damage first appears, particularly since the great tensile splitting forces there make cracks unavoidable and since, on the upper surface, temperature fluctuations induce large stresses. Research continues on a technical protection system that is materially as well as structurally absolutely foolproof. Currently a research program in this area is being conducted at the Federal Institute of Materials Testing (BAM) in Berlin.

CANTILEVER CONSTRUCTION WITH A TRANSPORTATION TRUSS

Even greater spans can be attained if the auxiliary truss does not carry the load of the concrete but serves merely as a transport bridge for materials, men and the form
carrier. With the same weight it can then span greater lengths. As in classical cantilever construction the already completed superstructure again assumes the load of the concrete (Fig. 27).

With a clever solution, the loading of the superstructure during construction and the increased need for reinforcing steel can be kept within economic limits. The cross-section is completely filigreed and freed from dead load, in that one first constructs a core cross-section that is reduced to the constructional minimum. A separate form carrier is subsequently added to complete the final structure (Fig. 23). Spans up to 150m are possible with this method.

The Kochertal Bridge in southern Germany can serve as a prototype. Fig. 24 illustrates the main starting phase at a pylon; easily visible is the light-weight transportation truss. The cross-section (Fig. 25) closely resembles the structures of steel bridge construction. The concrete sections were 5m long and construction proceeded at 10m per week. With a height of approximately 180m above the valley floor, this structure is at the present time Germany's highest highway bridge (Fig. 26). Prior to concreting the final panel, corrective section lengths were introduced (negative transverse force, normal compressive forces and a positive bending moment), in order to immediately establish a coupled, new system with complete
capacity for creep (penetration system) in individual section lengths, and thus to obtain only small movements.

Because of the self-weight of the added concrete and its creep and shrinkage, additional compressive stresses occur in the core cross-section, which are especially detrimental to the calculation in the area around the pylons. This situation creates a definite optimal criterion for completing the construction. Around 80% of the size of sections is a result of self-weight, the rest is to carry the traffic load specified in the German standards (Wayss und Freytag AG).

CANTILEVER METHOD FOR SEGMENTAL CONSTRUCTION

The push for ever-greater speed of construction led to development of the segmental construction method. In Germany, however, there have been few suitable projects that would give this method any overwhelming advantage over other processes. Fig. 27 shows an experimental model.

In the absence of real opportunities to test the method, doubts remain about the applicability of good experimental results to the practical construction of long-span bridges. The durability of the glued joints, the additional risks of injecting the spanning members, and their stress and strain as well as their fire resistance seem insufficiently proved to satisfy German perfectionism. So this method is still not generally permitted in the
Federal Republic. Neighboring countries, however, already have employed it successfully, especially France (Fig. 28). In formal discussions, this method is linked with that of pre-stressing without joints. The Long Key Bridge in the United States, recently completed, is a noteworthy example. Under the influence of the segmental construction method, which has since gained a foothold worldwide, this method will probably be permitted in Germany in a short time. To date, the fastest rate of construction has been 85m per week (on the Rio-Niteroi in Brazil); the longest cantilever span attained, 172m (Ottmarsheim).

In principle, all variations of cast-in-place construction methods are conceivable. Current views recommend the following criteria for the segmental construction method with extended joints (Guckenberger, Daschmerand Kupfer):

- complete prestressing,
- ignoring the tensile strength of the joint,
- development of special cement based mortar, and
- use of fine key in the web, with torsion in the entire cross-sectional area.

The demand for complete prestressing leads to increased use of reinforcing steel up to 50% with respect to
a partially pre-stressed girder of cast-in-place concrete. Total prestressing can be justified only if the advantages of the segmental method actually can also be exploited.

FORM CARRIER

The question of the form carrier running above or beneath the superstructure today has practically been decided in favor of the latter. Fig. 29 shows a carrier running above with transport trusses emerging from the front according to the slide rule principle. Its effectiveness is enhanced by freeing the bridge structure of supports. In contrast, the carrier can run underneath the superstructure (Fig. 30), a principle that offers the essential merits of bearing the load directly, support by pylon cleats, and a free working area on the bridge.

Another, newer, building principle (Fig. 31) eliminates the outrigger and substitutes a scaffolding carrier of flexible building elements. Steerable feeding and training prows allow direct support, curvilinear travel without special precautions and adjustments for each individual design. The loads on the cantilevered slab are, to be sure, somewhat greater, but the total weight is less. At the present time, the greatest distance between supports is approx. 65m.

Particularly advantageous is a support span of 35m. Beyond that, the carriers become extremely heavy and
uneconomical. An entire span length, from one point of zero moment to the other, is cast with concrete. Center support releases should comprise a span of 4.5 to 5.5m. In general the rate of construction is one bay per week. Fig. 36 shows a modern form carrier in use.

In all of Europe, a great number of bridges have been built using this method, which has proved very adaptable even in the case of objectively determined deviations from the standard patterns.

**FIELD-SPANNING PRE-CAST UNITS**

Although work has been and is constantly being done on developing a pre-cast building method, its use in German long-span bridge construction continues to be limited. Except for the erection of medium-span transport bridges around 20m long (Fig. 33), where the pre-cast method in conjunction with a cast-in-place deck often was able to capture a firm portion of the market, the pre-cast construction process has usually been confined to exceptional cases.

Surely the main reason is the required grid in the transverse and longitudinal directions, which, in part because of the overall design as well as the aesthetics, cannot always be accommodated. However, although forced submission has seldom led to convincing solutions, in no way should this drawback be misconstrued as a sign of the
inapplicability of this method. In the future we will surely need greater quantities of pre-cast units for noteworthy undertakings such as those depicted in Figs. 34 through 37.

**Step-Wise Shifting Procedure**

In the methods I have discussed the bridge is stationary and the casting site is mobile, whereas the step-wise procedure reverses this relationship and combines the advantage of casting in sections with that of an unmoving fabrication area protected from the weather. The bridge is pushed outward in longitudinal steps as if it were emerging from an extrusion press (Fig. 38). With very long-span bridges, intermediate feed units can be installed or the bridge can be pushed outward in sections, one from each side, that meet at center span. Bridges from 600 to 800m glide outward from the pylons (Fig. 39).

Concrete sections are 20 to 30m long, and one section is produced per week. A steel projecting prow reduces the bending moment. With longer spans, an auxiliary middle support should be provided (Fig. 40 and 41). For the path of the feeder, the superstructure is centrally pre-stressed (primary prestressing), while in its final position it receives further, secondary prestressing that resembles the moment diagram.
Normally, only spans greater than 150m are profitable. For smaller bridges less than 100m long a variation, the step-wise pulling process, was developed.

If the support action of an auxiliary pylon remains in the final system, then it is possible to suspend the superstructure from this point to produce a so-called stayed-girder bridge (Fig. 42).

**Rotational Procedure**

Often during construction, large clearances must be maintained without disturbing the traffic deck. For pre-cast methods the spans are much too great; for cantilever construction, the working distances too small. this in-between territory is filled by the rotational procedure. As of now, four such structures have been completed in Europe, an example of which is the Luchtringen Bridge over the Weser in Germany (Fig. 43) (Specht, Powitz und Friedigkel). Construction proceeds according to the following schedule:

**Phase 1:** Construction of supports on both sides of the river,

**Phase 2:** Construction of the first portion of the superstructure parallel to the river bank,

**Phase 3:** Rotation of the first part of the superstructure,
Phase 4: Repetition of the work on the other side of the river,

Phase 5: Rotation of second section of the superstructure,

Phase 6: Coupling of the 3-span bridge by means of a cantilevered concrete segment.

The spans were 46.5, 87.0 and 46.5; the bending moment was 30MN m (Figs. 44 and 45). The shortest distance between supports consisted of the necessary clearances plus one width of the bridge. The latter was not permitted to be too large.

Also in this category of methods are those such as transverse sliding and lowering superstructures that are poured overhead.

**Cable-Stayed Bridges**

Concrete structures can bridge very large spans by means of a rediscovered design: the cable-stayed bridge with inclined cables. (Fig. 46). As opposed to the auxiliary pylons of cantilever construction, the stay cables now are part of the completed load-bearing system of the bridge. Both the harp form and the fan form are used. After a few side-tracks during development, the following structural elements have come to be considered optimal (Leonhardt 1979):
The smallest possible distance is left between cables, so that the bridge can be erected as a cantilever without further temporary measures.

For each requisite point of support, there is only one cable, with a top service load not to exceed 10MNm. With two cable planes, there is a cable separation of 8-15m depending upon the width of the bridge.

The fan-shaped arrangement of the cables is technically and economically more advantageous than the harp shape. Two cable planes, sometimes anchored outside the cross-section, are technically better able to withstand vibration than a single centered cable, especially with long spans.

With a small distance between cables, the normal compressive forces are clearly dominant; the bending moments retire into the background. This allows an extremely economical cross-sectional shape, which now is governed primarily by the safety factor against buckling. Heights of 1.0 to 2.0m are possible.
- After discovery of the very effective system-damping of the inclined-cable bridges with narrow cable spacing (in contrast to suspension bridges), the shape of the cross-section is relatively free. Disruptive resonance vibrations are not possible (except for torsional vibrations of structures with one centered cable). Because of this excellent dynamic relationship, inclined-cable bridges are also well suited for carrying railroad loads.

- Only high-strength steels, such as reinforcing steel used for tensioning, should be used to achieve the greatest possible effective stiffness against elongation. Parallel wire cables of reinforcing steel are suitable (i.e., St 1325/1470 and St 1470/1670), drawn into polyethylene tubes (or steel tubes) and injected with mortar. Corrosion protection must be extremely dependable and must meet the highest standards. Absolutely every cable should be constructed so as to be replaceable; thus, each cable must have its individual anchorage.

- The cable's endurance strength against oscillation has been improved to 200 to 240N/mm² (Hi Am-cold capping), which is sufficient.
- The height of the pylons should be 1/4 to 1/5 the span width. Even in the area around the pylons, the bridge should remain hanging, and not become rigid.

- The most favorable method is cantilever construction with cast-in-place concrete using pre-cast elements in the anchorage area of the cables.

Both Figs. 47 and 48 show the Hoechst Bridge over the Main in Frankfurt, which carries traffic as well as railroad loads over a span of 148m. An American example is the recently-completed Columbia River Bridge in Washington State, spanning 299m. Leonhardt (1979) specifies maximum spans of 700m for road bridges and 500m for railroads.

**Highly-Prestressed Bridges**

Finsterwalder's highly pre-stressed bridges (1960) are especially fascinating design ideas. If reinforcing steel is distributed evenly over the width of the road deck, then placed over that from pier to pier, stressed excessively, and finally concreted in position, then the bridge is reduced to a directly travelled concrete deck only 30cm thick. However, extremely high prestressing forces and supports for changing cable direction are needed at each pier (Fig. 49).
Spans of 600m with a sag of only 1.50m between the boom ends have been designed. However, these bridges have dynamic problems and are very sensitive to vibrations. They depend upon an enormous expenditure of reinforcing steel and require favorable ground conditions for anchoring the horizontal ties (around 800MNm). So currently there are merely three modest structures in Europe; they are foot bridges. In Fig. 50: the bridge in Freiburg with an approximate span of 40m and deck 25cm thick.

Cross-Sections Offering Noise Protection

In countless cases, elevated highways must be constructed through the center of an already-inhabited area. Affected citizens today demand built-in protection from the resulting traffic noise. The design engineer cannot ignore this demand; so recent attempts have been made to design noise-protective cross-sections for use in cities.

Figs. 51 and 52 show the structure of an elevated highway in Dusseldorf. This development is still in its initial stages. A further example is the work of Engineering Candidate Steffan Both, a student of the author. A few weeks ago this research was honored with the Award of Recognition from the Society of Architects and Engineers (Fig. 53).
Arch Bridges

This final section deals with the most venerable of all bridge types: the arch bridge. The stone bridge in Switzerland shown in Fig. 54 displays the aesthetic attraction of the arched form. In Sweden, the Sandesund Bridge (Fig. 55), completed in 1940, was one of the greatest engineering achievements of its time. With a span of 264m, it remained for over two decades the longest-spanning concrete bridge in the world. The clear-span falsework, which was ultra-modern at the time, collapsed under half the concrete load. Despite this mishap, the bridge was finished. Germany possesses several beautiful arch bridges dating mainly from the beginning of superhighway construction. Each one was built with conventional falsework. Fig. 56 is an example.

After the war, construction of arch bridges stagnated. These structures could not keep pace with the modern bridge systems that had been developed in the meantime. Only with exceptionally favorable topographical conditions did the arch at times come under consideration. Even today, the arch is limited (with individual exceptions) in Europe to the Alpine region and to sea arms with fjords like inlets. As a rule, the efficiency of laminated wood construction is utilized for cantilevered falsework (Cruciani's method) (Fig. 57).

More recently, there has been a renaissance in arch bridge construction, as newer approaches have been
rationalized with the older, tried and true, methods. The first step was to build the arch as a cantilever with fan-shaped cable stays. Longer-span bridges have been supported with several fans (Figs. 58 and 59).

Modern sliding and climbing formwork has significantly reduced the cost of producing the carrier piers; finally the carrier girder could be pulled along economically by means of pre-cast units or the step-wise construction process.

Backwards inclination of the half-arch, which enjoyed a temporary vogue, soon ran up against practical weight limits. A meaningful step forward was recognition that the arch, piers, and deck form a unified system. An additional, diagonal, tensile element complete the stable truss system, which henceforth can be erected as a consoled, free cantilever from both river banks. The overview in Fig. 60 depicts the integrated construction method, for example:

- cantilever construction of the arch,
- fan-shaped auxiliary cable stays for the arch parts between flange joints,
- sliding formwork for the piers,
- cantilever construction with auxiliary cable stays for the carrier girder.
The Japanese designed such an arch bridge, spanning 170m, in which they left the tensile elements as part of the final structure (Fig. 61).

The segmental construction procedure may be allowed in the near future, especially for the standing arch always in compression, because the disadvantageously great need for reinforcing steel in arch structures can be reduced drastically and an excellent series of bridges can be created.

In 1980 an especially impressive result was attained by the Yugoslavs, who, with this principle—but with the tensile elements used only as temporary auxiliary structures—built the longest-spanning arch bridge in the world at that time. It spans 390m (Fig. 62 and 63). The building profession has only recently achieved large spans with arch bridges, a development that was foreseen by Leonardo da Vinci 477 years ago, when he designed his arch bridge over the Bosporus with a 240m span (Fig. 64).

**Outlook**

In the future, the development of concrete bridge construction will be shaped by the need to lower onsite labor costs, to divide the total effort into as many similar operations as possible and to mechanize them, to make molds for different-sized building elements, and to produce only prefabricated, quality-controlled building materials. The
cross-sections of bridges with normal spans can hardly become thinner. In contrast, with respect to column spans near to the ultimate bearing distances for specific methods, the main goal is a radical reduction of dead weight. Often, concrete with lightweight aggregates has been hailed as the material of the future. Because of its unfavorable cost structure and reactive attributes (shear range, tensile strength), it has not influenced concrete bridge construction significantly up to now.

The development of concrete bridge construction, beyond what is possible today, cannot be foreseen. Despite the currently high state of technical possibilities, experience has proved that various areas still are open to improvement.

For example, in the structural area, would be desirable for research to lead to simplified reinforcement. Also needed is complete closing of the gap between the nonprestressed method and reinforced concrete, and a complete freedom in selecting the degree of prestressing from 0 to 1. In addition, the positive forces and their effects should be researched to such an extent that finally structural and technological rules can govern them. The crack problem should be eliminated in its entirety with regard to causes and manifestations, but the correction of the crack problem has yet to be solved completely.
Durability and maintainability present a large problem. During cost estimating of bridge designs, ease of inspection and of maintenance play an ever-increasing role. Further experiments involving the behavior of current bridge systems during an earthquake and the possible consequences for the designers should be conducted.

No longer can the engineer optimize individual aspects, such as the structure or the design, but must optimize all performance factors simultaneously. In the future he also will have to consider the maintenance costs and include them in the final balance. For that the designing engineer requires more breadth. Depth of knowledge - yes; narrow-mindedness - no. Researchers can see themselves as specialized advisors in this process.
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CHAPTER TWO

SOME CONSIDERATIONS IN THE DESIGN OF CABLE-STAYED BRIDGES

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Though a cable-stayed design for bridges was first proposed over 100 years ago, they have only been important in roadway construction since 1950. Before then, engineers had found it difficult to calculate the forces of those static systems with high degree of indeterminacy, and construction technology had not advanced to the point of producing tension elements that were strong enough to guarantee that bridges would be sufficiently rigid.

To combat the problem of overly flexible decks, some of the earlier cable-stayed bridges incorporated features of the suspension bridge into their designs. The Albert Bridge in London (Fig.1) is one of the oldest structures of this type. The straight cables are the ones carrying the main girders. The curved cables running to the tops of the towers have hangars, a characteristic of suspension bridges. This structure also exhibits the intricate ornamentation that was beloved by designers in the Victorian Age.

The full potential of cable-stayed construction has
yet to be realized. Modern spans of this type range from 200 to 600 m. But according to F. Leonhardt, an eminent bridge designer, future spans will be able to reach 1700 m. Thus, this design principle eventually can be used not only for medium but also for long spans.

Compared with other bridge systems, the cable-stayed design is economical and has additional advantages:
- The longitudinal girder has a smaller depth than that of a beam bridge.
- The bridge undergoes smaller deflections than beam and suspension bridges.
- Its overall dynamic behavior surpasses that of either beam or suspension bridges.

Arrangement of the Cables

Of the four standard arrangements (Fig. 2), the fan is technically and economically the most feasible. Because the cables in the fan shape are inclined at a steeper angle than those in the harp shape, the load is concentrated at the cable anchorages and thus produces lower forces in the cables and the deck. This configuration also requires less steel for the cables.

The fan and the harp have anchorages distributed over both the height of the tower and the length of the deck. With the other two arrangements, anchorage points
are concentrated and thus difficult to construct. In the brush shape, cables join at the head of the tower; in the star-shape, they are connected together at one point in the main girder.

Beyond engineering considerations, aesthetics influence the choice of a cable arrangement. When viewed from a skewed angle, the cables in the harp shape appear to be parallel, which makes it visually appealing. This particular arrangement was chosen for a family of Düsseldorf bridges crossing the Rhine (Fig.3). Other bridges use different cable configurations (Figs. 4,5,6,7).

From time to time it is necessary to replace the cables, consequently, design should facilitate replacement. Distributing the cables over the height of the tower proves simplest. An arrangement with all cables joining in a point at the top of the tower presents difficulties in replacing a damaged cable. Thus, either the fan or the harp should be used, though other configurations are possible. The determining factor is the ratio of lengths between the main and side spans, which differs with each design.

In general, cables can be placed in either on plane or two. With two cable planes (Fig.8), fewer cables are needed, and it is possible to use arrangements other than the fan or the harp. Figure 8 also shows how the slope of the cables is determined by the ratio of the
main effective span \((l)\) to the tower height \((h)\).

When the cables are in two planes the anchorages can be placed either inside or outside the bridge railing. If they are inside, the bridge must be widened to accommodate the towers and the cables, and this creates dead space that cannot be used to carry traffic. In the second instance, auxiliary constructions are needed for transmitting the cable forces from outside the railing onto load-bearing cross sections (Fig. 9). This design also requires piers on the towers.

Normally the tower columns are nearly vertical, so that no horizontal forces occur in the cross direction. When an A-shaped tower is chosen, often on aesthetic grounds (Severin Bridge), remarkable cross-directional horizontal forces arise.

If the structure comprises two separate roadways with a space between them, a single plane of cables is preferable. The load-bearing construction is then concentrated above this central space, and traffic lanes are not eliminated. Figure 10 shows four bridges that use one place of cables in various arrangements. An advantage of the single-plane construction is that the pier width depends only on the width of the longitudinal girder (Figs. 11 and 12). But it is disadvantageous that the longitudinal girder has to be a box beam, so that its torsional rigidity is great enough to bear eccentric loads.
The number of cables should be chosen so that they do not have to be specially anchored, and so that auxiliary constructions are not needed during erection. With only a few cables, the internal forces are so high that each strand of every cable has to be anchored separately, which is a rather expensive process. If the cables are spaced too far apart, auxiliary constructions are required during erection for the free cantilever sections that reach the next anchorage point.

Because low bending stiffness decreases the bending moments due to live loads, the depth of the longitudinal girder must be chosen to satisfy the requirements for deck buckling safety when the bridge is under longitudinal, perpendicular compression. If a structure has only a few cables, the bending moments in the main girder are kept small, and thus the depth of the girder must conform to the regulations governing heavy traffic. Curvatures must remain within limits.

The main girders must be at least as deep as the 2m cross girders. If cables are spaced 10 to 20m apart the main girder can be from two to three cm deep. To achieve cable forces of a uniform magnitude in all regions, the anchorages must be spaced closer together in the mid-span of the bridge than in the support zone.

A major disadvantage of a large number of cables are the correspondingly numerous points of connection
(Figs. 13 and 14). Each of these anchorages must be specially constructed.

For a static analysis, the multi-cable system can be approximated as a beam on an elastic support.

Cross Sections

In cable-stayed bridges, the cross sections must provide the torsional rigidity that the cables cannot. Usually, T-beams and box girders are chosen as sections (Fig. 15), and normally the deck is constructed as an orthotropic steel plate. With one place of cables, a box girder is the only possible choice for a cross section. Some bridges with two cable planes also require a box girder in order to counteract heavy, eccentric loads, such as those from a subway (Fig. 15).

When there are only two main cables, a box girder of greater depth than normal (4.2m in Fig. 16), is needed to provide sufficient resistance to torsion. In Fig. 16, inclined columns spaced at 5m support the cantilever arm of the orthotropic plate. Lindner (1981) has discussed the advantages of the hollow sections used in the ribs of the plate.

Leonhardt points out (1980) that for multi-stay cable bridges, vertical bearings should be placed only at the ends of the side spans, not at the tower. As seen in Fig. 17, the flanges of the Leverkusen bridge pass
through the cross girder to the rigid support on the pier. To make this connection conform to Leonhardt's principle, the elastically deformable support condition given by stay cables should have continued. Stiff, vertical supports constructed at the tower induce large, longitudinal bending moments, which decrease the bending stiffness of the bridge below the allowable limit.

Cables

In bridge design, careful attention must be paid to determining the modulus of elasticity of the cables. The modulus differs with each manufacturing method. In Germany, the cables are normally made as helical strands of high-strength steel wires. Though prefabricated, parallel-wire strands are also available, they are used in only a few cases, largely because they offer better resistance to corrosion.

Conversely, the larger modulus of elasticity for prefabricated, parallel-wire strands gives them an advantage over helical bundles. Although parallel strands formed with a spinning wheel usually are too costly and too time-consuming to be used on multi-stay bridges, structures with few cables are able to use them. In this method, individual ropes are fabricated from several wires and then gathered into a single cable (Fig. 18). To protect these cables, laborers must work from an auxiliary
footpath.

Static calculation of cable forces can be made with tolerable effort and cost if the cable is heated as a bar. By introducing the term $E_i^*$ the reduced stiffness of a long cable owing to its sag from self-weight can be accounted for. From the evaluation of the differential equation for the cable we get (Ernst 1965):

$$E_i^* = \frac{E_s}{1 + \frac{E_s}{12} \frac{1 \cdot x^2}{1^3} \cdot 10^8}$$

Taking $I_m^*$ as the mathematical average stress, which is proportional to the value $I_o + \Delta \sigma/2$, $I_o$ as the prestressing and $\Delta \tau$ as the stress factor from the live load, we can see the behavior of the system. The stiffness of the cables increases with the third power of the steel stress, and decreases with the second power of the horizontal length that the span is increased due to sagging. Results for helical bridge strands are shown in Fig. 19. Note that $I_m^*$ and 1 heavily influence the results.

The rigidity of $E_i^*$ can be reduced somewhat, but only for cables of great length if they are made of high-strength steels amenable to significant prestressing. Such steels are now available. For example, Germans use a steel with ultimate strength of
\[ I_Z = 143 \text{ kN/cm}^2 \]

According to DIN (the German National Standards), the allowable stress then becomes

\[ I_a = I_Z / 2.2 = 65 \text{ kN/cm}^2 \]

If the prestressing is chosen as

\[ I_0 = 40 \text{ kN/cm}^2, \]

the stress for the live load remains

\[ \Delta \tau = 25 \text{ kN/cm}^2. \]

Evaluation of this formula leads to

\[ I_m^* = 47 \text{ kN/cm}^2, \]

an average stress that takes the sag into account. From Fig. 19, we get for \( l = 300 \text{m} \):

\[ E_1^* = 15,600 \text{ kN/cm}^2 = 0.92 E, \]

yielding a reduction in rigidity of only 8 percent.

Because the cables are the most important members of
this system, they must be of the best quality and must be safeguarded against fatigue and corrosion. Test results and practical experience have led to the development of several types protective measures.

In the first large cable-stayed bridges, steel ropes protected only by paint caused problems. To secure the cables from corrosion, they should be placed inside a tube of polyethylene, which can keep its properties for at least twenty years (Leonhardt, 1980). Alternative materials for the tube include steel, preferably stainless, or any other metal that can be easily protected by painting.

The fatigue strength of the cables depends on the fatigue strength of the anchorages. Normal, zinc-filled sockets in the steel ropes do not yield the greatest fatigue strength, because the high temperature required for pouring the zinc damages the strength of the steel. Therefore, special anchorages have been developed, to employ a cold filling material in the sockets at the ends of the cables.

Traffic causes unavoidable damage to bridges. Thus anchorages should be designed to allow easy accessibility to individual cables, should they need replacing. In addition, built-in hydraulic jacks can regulate the length of the cables as needed.
Cable Anchorages

Because they are subject to the cable forces, anchorages often have to be designed specially for each job. If necessary, these connections can transmit the cable loads over special cross and longitudinal girders to the main girder. Depending on the arrangement of the cable plane, the anchorages lie either inside or outside the main girder.

Expansion of cable anchorages often permits a bridge to be designed with few cables and high cable forces. Such an expansion enables the individual ropes to be anchored separately (Fig. 20). A cable clip, usually about 15m long and 0.5m high, holds the cable in place (Fig. 21), while the cable ends are supported by thick plates that transfer the cable forces into the vertical beams (Fig. 22). From these beams, the forces are led over thick cross girders into the web of the main girder.

Pylon

Cable Anchorage on the Tower.

There are two ways to anchor the cables on the tower: they can be left to move with respect to horizontal forces, or they can be fixed.

If horizontal motion is to be permitted, difficulties arise in constructing the movable anchorage, because normally, movement between the cable and the substructure is
excluded. When the cables are continuous and have to change their inclination within the tower, a cable saddle is used (Fig. 23a). Thus the cable is supported on roller or sliding bearings. An overturn bearing (Fig. 23b) is another possible means of providing a movable support. In this instance, the bearing has a sectorial form about 2.5m high. As it rotates around a strong bolt of St60, it rotates with a diameter of more than 2m, and corresponds to the behavior of a movable bearing.

If the cables are fixed, a set of different horizontal forces is guided into the pylon, which causes additional longitudinal forces bending moments into the pylon. But here the supports for the main girder are more rigid so that they provide a better moment distribution than does a system with a movable support.

To construct a fixed support against horizontal forces either the cables can be jointed at the tower of they can be continuous. Reasons for using jointed cables (Fig. 24) include: relatively small cable forces, large angle for redirection, and small difference in height between the individual cable planes. Crossing the cables at the face of the tower and securing them with anchors is another solution. But here, unbalanced eccentricities perpendicular to the longitudinal direction may cause the tower to twist.
If the cables are continuous, cables saddles can be used for regulating and adjusting the forces in them. The saddles must allow for horizontal and vertical displacements and require auxiliary constructions for placing the presses (Fig. 25). Often it is better to adjust the cables at the anchorage points on the main girder in order to avoid excessive jacking of the cable saddles.

Though the shape of the tower is sometimes subject to special architectural treatments, engineers should try to keep the forms as simple as possible. Tapering, good proportioning and suitable cross-sectional profiles aid in achieving a pleasing appearance. In the case of a bridge over the North Elbe at Hamburg (Fig. 26), the upper part is nearly unloaded - only dead weight and wind forces cause stress. The A-shaped tower of the Severin Bridge (Fig. 22), seems somewhat heavy in comparison. In Hamburg, the towers of the Köhlbrand Bridge (Fig. 28) look much more pleasant and serve as the gateway to the port for industrial traffic.

The inclination of the tower for cable-stayed bridges is worthy of notice. Some structures, such as the bridge across the Danube in Bratislava, Chechoslovakia, have been built with the tower inclined backwards. Though this orientation makes the back stay shorter and steeper, the bending moments of the tower increase correspondingly. Thus, this form is not economically advantageous and is
technically more difficult than a straight tower.

An alternative to erecting a backwards-reposing tower is constructing a large angle between the cable and the main girder. This arrangement easily transmits vertical loads and reduces the forces in the rigid cables that provide elastic support. In addition, the horizontal components of the compressive force on the main girder are reduced. On disadvantage of this method is that a steeper cable incline requires a higher tower, which is then subject to greater bending moments from the horizontal forces. This then leads to larger overall dimensions for the tower.

The method of anchoring cables on the tower greatly influences the dimensions. A taller tower is needed to accommodate relatively large cable saddles, because there must be a certain distance between the saddles. Arranging fixed anchorages on the tower without redirection reduces the spacing between cables. This is another reason for using multi-stay cable systems in which the individual cable forces are not so great. Although this design entails relatively low costs for the cables and main girder, it raises the cost of tower construction. Whether an overall saving can be achieved obviously depends on the net cost of the whole system.

Aesthetics play a large part in the choice of tower height, because the finished structure should be inte-
grated into the landscape. In the bridges illustrated, the ratio of maximum span to tower height lies between 3 and 6. Multi-stayed bridges are generally designed with values between 4 and 5.

Dimensions of the Tower.

The tower has to bear large compressive forces and moments, so it is much more economical to transmit the unbalanced components of these forces from the top of the tower to the ground via the back stay cables than to increase the bending resistance of the tower. Because buckling occurs both lengthwise and cross-wise, a box beam girder is preferable to other shapes. For example, the girder in the bridge over the Rhine at Düsseldorf-Oberkassel is 4.2m deep and 3.0m wide. If the tower is slender and has a low bending stiffness, live loads in the main span do not cause large bending moments in the tower. In the same bridge, the steel plates are, at a maximum, 70mm thick and are made of steel grade St 52. The construction firm tries to use high strength steel, with a yield point of 700 N/m², which reduces the dimensions and requires lower design rigidities. Though these changes decrease the bending moments resulting from the different cable forces, it also decreases the stability of the structure.

Any cable-stayed bridge must meet two criteria: the
tower should be flexible enough to absorb the deformations transmitted by the cables, and it should have necessary buckling rigidity. Although a rigid connection does not allow the tower to handle the displacements caused by the live load, it does support the tower against buckling along the depth of the bridge.

When concrete instead of steel towers are used (as in some bridges built recently in Germany), box sections are still preferable to solid cross-sections. Box sections allow access to the cable anchorages from inside the tower shaft (Fig. 29) and slits in the tower columns save concrete (Fig. 30). The base of these slits also provides an easy support for the cross girder at the tower section.

**Static Calculations**

By carefully choosing the pre-stressing forces in the cables, the engineer can generate a nearly constant bending moment in the main girder. Consequently, the cross section of the girder often can remain constant throughout the length of the bridge. These are the significant differences between cable-stayed and other bridges. The larger the number of cables that are available for "manipulating" the bending moments, the easier it is to control the design of the bridge.

The traditional method of setting up and evaluating influence lines sometimes has been replaced by calculations
of partial loads (Beyer and Lange 1974). Leonhardt (Beyer and Lange 1974, Leonhardt, 1980) made extensive comparisons of the two methods (Figs. 31 and 32). The most favorable range for cable-stayed bridges is a main span \( l \) to tower height \( h \) ratio of 6 to 4 (Fig. 31). In comparison with suspension bridges, fan-shaped cable-stayed bridges are more economical, especially as span length increases.

**Erection**

Cable-stayed bridges are erected in much the same manner as steel bridges. The newest Düsseldorf-Heke bridge is an example (Kahn 1979). First, the A-shaped tower with a capping vertical shaft was constructed (Fig. 29). It is able to withstand the large forces. Over 15,000 m\(^2\) of bridge surface are supported by the tower via the cables. Though it was impossible to build a vertical pylon in the middle four meters of the bridge's width, aesthetic reasons for a single plane of cables dictated the form of this tower. To save concrete, vertical slits were made in each column, to form a channel section (Fig. 30). These slits decreased the dead weight and lowered the longitudinal stiffness of the tower.

In this structure, a cross girder at the tower supports the main girders (Fig. 33), composed of five
box girder sections. The joint to the side-span bridge of concrete is illustrated. At the inclined right column of the tower is a wider slit that accommodates the cross girder.

A few weeks after the tower was poured, the box beam was connected to the concrete bridge (Fig. 34). In this arrangement the normal and vertical shear forces are transmitted by headed studs to the walls of an additional steel bow. From these walls, the forces travel to the supports in the tower columns and to the steel bridge at the opposite end of the cross girder.

Somewhat later, the concrete bridge was finished and the steel bridge was in its first phase of construction (Fig. 35). Auxiliary supports were used to hold up the first 15 m. The cross girder was tapered at its end to fit into the slit in the tower column. And because a large bending stiffness is needed only at midspan of the cross-sections the ends need not be as large.

A three-celled box girder forms the cross-section of the steel bridge. The middle cell is welded from four plates, two for the webs and one each for the upper flange and the lower flange (Fig. 36). The ribs for the deck plate consist of hollow trapezoidal sections. The same cross section is used for stiffeners in the webs and in the lower flange. Because some cables are anchored at this connection between the steel bridge and the concrete
bridge, the intermediate portal frame is very rigid. To protect it from corrosion, the cross girder was galvanized. Once welded, this cell was hoisted to the top of the bridge and then jointed into place (Fig. 37).

The middle cell is connected to the cross girder and to the other parts of the two side cells (Fig. 38). Plates for the upper and the lower flange complete these cells. The view in the figure is from the deck plate of the middle cell to the far wall of the side cell. Throughout its length, the bridge is erected as a cantilevered construction.

Cable sets consist of 6 individual strands (Fig. 39), in three layers of two strands each. This allows each strand to be anchored separately, and to be replaced easily. The force in the cable set is one-sixth of what it would be if all six strands were encased and anchored as one.

Cable anchorages on the deck plate need special protection from water (Fig. 40). On the left side of the figure is the protruding end of a tube that will protect the cables against corrosion potentially caused by the salt used to clear bridges of snow. The tube is connected to the deck plate but there is no rigid connection between it and the cables. This ensures that the deformations at the cross girder between the deck plate and the cable anchorages will not affect the cables.

Heavy rings transmit forces to the cross girder, which then transfers them to the webs of the middle cell of the
main girder. A jack is used beneath the anchorage points to post-tension the cables (Fig. 41).

After it was completed, this new cable-stayed bridge over the Rhine was nicely integrated into its environment. This aerial view of this Düsseldorf bridge shows the beauty and elegance of the simple harp configuration (Fig. 42).
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FIGURE CAPTIONS

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Figure 42: Aerial view of the new Düsseldorf-Flehe Bridge over the Rhine.
CHAPTER THREE

CONSTRUCTION AND ANALYSIS OF ORTHOTROPIC STEEL PLATE DECK BRIDGES AND COMPOSITE BRIDGE DECKING

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The main girders of a bridge transmit traffic loads in the longitudinal direction. The girders receive these loads from the bridge deck, which may be constructed of orthotropic steel plates or concrete slabs.

If the deck is to be an orthotropic steel plate, cross girders are necessary for reducing the amount of material and the span length of the floor slab. Additional longitudinal ribs may be placed between the cross-girders and parallel to the main girders (Fig. 1).

An example of this type of construction is seen in a bridge spanning the Danube at Regensburg (Fig. 2). Here the main girder is a T-beam. In the area of the cantilever arms and between the main girders, the cross-girders are spaced 5m apart. Running parallel to the main girders are the hollow-sectional ribs, which are spaced even closer.

Since the cantilever arm of the girder as originally designed was too wide, inclined columns were used at every second cross girder (Fig. 3). The cross girders and the ribs are located between the two main girders,
except in the center region, which is the space between the two roadways (Fig. 4).

Previous bridge designs have attempted to treat the structural members individually, and bridges have often been constructed so that the members did not interact statically. But a steel deck plate is economical only if it is part of an integrated system in which all of the members interact with one another. Thus, the plate not only distributes loads to the main load-carrying members, but it also acts as the top flange of those members.

In this orthogonal-anistropic system, abbreviated as orthotropic, the steel deck and the ribs can be regarded as a plate. The other parts of the structure can be considered as a girder grillage. Thus the plate exhibits different rigidities in the x and y directions (Fig. 5), which are perpendicular. In this steel bridge, the condition is an anisotropy due to construction as opposed to material as in wood.

To calculate load effects, sub-structures may be considered separately (Lindner 1980; Lindner and Bamm 1982), and stresses from the sub-structures are superimposed (Fig. 5). For instance, in this classification, rib stresses may be computed from Systems II, III and IV, and the plate stresses from Systems I, II, III, and IV.

Theoretically, the plate should be designed for the
maximum bending stresses that result from superimposing of all the bending stresses. But in practice, this is not necessary. In Germany, if the deck length is less than 30 cm, the bending stresses from System I may be neglected. This practice is supported by tests which have proved that there is a low probability of the yield stress occurring, and by the fact that the deck plate as a membrane has a large reserve strength and is kept within the elastic range.

Construction of the Orthotropic Plate

Although the ribs in the plate can run in either the longitudinal or transverse direction of the bridge, they normally run in the longitudinal direction and thus act as part of the top flange of the main girder. This particular system is very efficient at resisting bending because the ribs and the main girder function as a single cross-section. The neutral axis of the section is in its upper part, and the maximum stress in the deck plate is relatively low.

In contrast, if the ribs run in the transverse direction, they do not act in the cross section of the main girder. With this arrangement the stresses are greater (Fig. 6) and special attention must be paid to ensure stability against buckling. With ribs running transversely, cross girders are unnecessary and the construction
of joints can be easy. But because of the higher stresses, this method of construction is used only for small loads (Fig. 7).

The ribs may have either an open or a closed cross section (Fig. 8). Open rib sections such as plate, angle, bulb, and channel beams are easy to fabricate and to splice in the field. Though they allow access to the underside of the deck for inspection and maintenance, open sections have little torsional rigidity and thus should only be used for spans under 3m.

In contrast to the open cross-sections, the box beam sections such as channel and box beams have great torsional rigidity. Since they can span up to 6m, less steel and fewer welds are required than for open-rib decks.

Bridges in Europe have used several different rib sections for orthotropic steel plate construction (Fig. 9). In the first two diagrams are sections with bulbs, which originated in ship design, and plates with a span of 2m. The third diagram is of a special profile that was welded from half beams and plates, and that spans up to 3.6m. In recent years, increasing welding costs have made this profile uneconomical. The channel section shown in the fourth diagram is not welded, and can span up to 4m. In the last diagram is the trapezoidal section that has been most used in recently constructed bridges. This section has been favored because it has a large moment of inertia.
and can be fabricated easily from plane plates. Spans of 5m are common for the trapezoidal section.

Analysis of the Orthotropic Plate

Normally, the steel plate is analyzed as a continuum, according to the differential equation derived by Haber in 1923 (Fig. 10). Although it is assumed in the derivation that the \(x\) and \(y\) axes are in the same horizontal plane, in actual constructions this is not the case. However, the results from the differential equation are sufficiently accurate. The main parameters of the expression are:

- the flexural rigidity, \(B_x\) and \(B_y\).
- the effective torsional rigidity \(H\), composed of the torsional rigidity of the covering plate (B) and the mean value of the rigidity of the ribs in both directions (\(H^*\)).

For special cases, the solution of this differential equation is available. But for isotropic plates, the following substitution is made:

\[
H = \sqrt{\frac{B_x B_y}{2}}
\]

even though this condition normally is not fulfilled.

Because of the inadequacy of the Huber equation, German engineers frequently use the solution of the Pelikan/Esslinger
differential equation (Fig. 11) (Pellikan and Esslinger 1957). The solution for open cross-sections is simplified with the following:

$$H = \frac{K_x}{x} = 0$$

This substitution reflects the torsional rigidity of the plate itself and the flexural rigidity of the plate in the direction normal to the ribs. Thus the moments depend only upon the span length of the bridge. According to the German standards, the point loads that must be taken into account occur either at midspan or at the columns (see Table in Fig. 11).

For hollow cross sections, the solution is simplified if

$$\frac{K_x}{x} = 0$$

is assumed. This substitution neglects the flexural rigidity of the plate in the direction normal to the span, and the solution also depends on the ratio $\frac{H}{K_x}$ (see graph in Figure 11). Although the Pellikan/Esslinger equation is just an approximate method, it requires long, tedious computations that are best left to computers.

Giencke (1955) introduced an additional refinement with an equation that takes into account the existing eccentricity, $e$, of the neutral $x$- and $y$ axes. With this
method, the calculated torsional rigidity increases and thus the deformations and the stresses in the deck plate are reduced.

To simplify for practical use, Gienckes expression has been evaluated for certain hollow sections (Lindner 1980, Lindner and Ba mm 1982). The loadings taken into account are those defined by German national standards (Figs. 12 and 13). The most important parameter, the depth of the ribs, \( h_r \), was varied from 200 to 350 mm; the thickness of the ribs, \( t_r \), was chosen as either 6, 8, or 10 mm; and the thickness of the deck plate, \( t \), was 12, 14 or 16 mm. Also considered was the angle \( \alpha \), the ratio of \( h_r / t_r \).

From Fig. 12, the stresses in the middle of the deck plate and on the bottom edge of the ribs can be read for the moment at the support as well as for the moment at midspan. All values are given for cross girder distances from 2 to 6 m.

The reduction factors, \( \psi_f \), for the moment at midspan and \( \psi_s \), for the moment at a support, can be determined from Fig. 13. These factors are multiplied by the moments, which are calculated for simple beams with rigid supports. Again, this evaluation is done using the single load that is by the German National Standard for the bridges that to carry the heaviest loads. Significant differences occur between the full lines, which are valid
for the moments at midspan, and the dotted lines, which apply for moment at a support.

The moments at midspan are much affected by the distance between cross girders. As this spacing increases, \( v_f \) decreases, which means that the adjacent ribs bear a significant portion of the load. In this situation, the transverse component of the load is well-distributed.

**Calculation of Girder Grillages**

Although computers are needed for complete calculation of girder grillages, simple procedures are preferred for estimations and preliminary designs. These methods are basically used to determine the transverse distribution of the loads (Fig. 14).

For example, if one of the three main girders is loaded, deformations occur. When this girder is not joined with the other main girders, the deformation is unrestrained, meaning that a load positioned about girder No. 1 is borne by this girder alone, and the influence factor is 1.0.

But if the connection between main and cross girders is rigid, the first girder is even more deformed and the influence factor, \( n_{11} \) is less than 1.0. The other main girders also experience deformation \((n_{12}, n_{13})\) which means that they help transmit the load. The transverse distribution, and therefore the girder-grillage effect, compensates for deformation differences. These occur only
with concentrated loads, for example, with wheel loads that follow the standard for heavy trucks. With equally-distributed loads, all main girders deform equally, so that there is no girder-grillage effect.

For infinitely rigid cross girders, the simplest equation for calculating transverse distribution has been formulated Engesser (bottom of Fig. 12). If the rigidity is taken into account, the parameter $z$ is used (Fig. 15) (Lindner 1980). This factor is large when the length of the girder, $L$, or the flexural rigidity, $I_Q'$, is great.

For simple girder grillages with three main girders and one cross girder, the solutions using $z$ coincide with those if Engesser, who assumed a flexural rigidity, $I_Q'$, of infinity. At the bottom of Fig. 15 the transverse distribution for main girder No. 2 is shown. With the very small value of $z = 0.16$, the load has no transverse component, so that the flexural rigidity is nearly 1.0. With the very great value of $z = 4300$, all main girders assume an equal portion of the load, so that the flexural rigidity is 0.33 for each girder. As Fig. 15 illustrates, the girder grillage effect is significant only for $z > 5$.

The number of cross girders has very little influence on the transverse distribution (Fig. 16). Increasing the total from $n = 1$ to $n = 5$ improves transverse distribution for main girder No. 2 by only 15%. Other procedures, for ins-
tance those of Leonhardt-Andra (1950), are based on these considerations.

From tables, one can read the figures for the transverse distribution in relation to the rigidity factor, \( z \). Once the distribution is known, the calculation for dimensions of the main girder and cross girders can be done using the usual methods.

**Construction Details**

There are two generally accepted methods for manufacturing the orthotropic plate (Fig. 17) (Weitz, 1979). In the first method (top of Fig. 17), the ribs are fitted between the cross girders, which have a complete cross section. Then the ribs are welded around the web of the cross girder. In the second method (bottom of Fig. 17), the ribs are continuous and the cross girders are notched where they meet the ribs.

With the second method, there are fewer manufacturing problems because the tolerance can be adjusted at the joints of the ribs. Conversely, the cross girder is weakened so that the transfer of the shear forces in the weakened web is difficult.

An example of continuous cross girders with ribs welded between is seen in a new bridge across the Danube near Regensburg, West Germany (Fig. 18). To increase fatigue strength, sometimes the ribs are welded only at the bottom
flange of the cross girder (Fig. 19).

With an open rib to avoid crossing welds from the deck plate and the rib, it is necessary to move the joint in the deck plate longitudinally about 50 mm away from the joint of the rib (Part A of Fig. 20). The small plate lying underneath the deckplate is welded on one side in the workshop, before it is shipped to the site.

There are two methods for constructing the joints of hollow ribs. In the first method, the deck plate is connected as described above, and the joint of the ribs is constructed with a metalplate, which is welded at the construction site to both sides of the rib (Part B) of Fig. 20). With the second method, the joint of the hollow stiffeners is made from the middle 300 mm of the rib. This actual length is determined by on-site conditions.

Fig. 21 shows an example of the joint of a plate rib that was constructed with a weld. Here, the joint in the rib and in the deck plate are stationary. One of the plates is a drawing of the type of weld used.

Shown in Fig. 22 is a joint of the hollow ribs using a middle section of the ribs (not yet in place). The upper plate, illustrated in Fig. 22, shows a joint between a hollow cross-section and an open one; the joint is necessary because the two ribs are of unequal depth.

For transporting the prefabricated parts, the entire cross-section of a bridge must be divided into sections.
For example, the bridge in Fig. 24 was constructed by parts. Its maximum span is 220m, and it is 10 meters high at the support and 5m at midspan. Fig. 25 shows the cross section and its components. The main girder consists of two parts; belonging to the upper part are two small portions of the cross girder (2700m long). The orthotropic steel plate of the inner region is brought to the field in five parts, but then is welded together on the construction site. At the cantilevers, a 3575m wide plate and then sidewalk complete the cross section. The sidewalk is used to correct for the curved appearance of the bridge.

The main girder is first to be constructed. Then, the adjoining plate is erected. Fig. 26 shows a longitudinal splice joint of the main girder and the lower flange. High-strength bolts are used in all of these joints.

Fig. 27 shows the lower part of the main girder at the site of prefabrication. The bottom flange is also stiffened with hollow ribs. In Fig. 28, the upper part of the same bridge is seen at the prefabrication site. Visible are the hollow-cross sections of the ribs for the orthotropic steel plate, and one part of the cross girder. To the left of the web is a rib that is larger than the other ribs, because the transverse distribution for it is not as good.
Composite Bridge Decking

Of the two materials used in steel composite construction, steel and concrete, steel has a greater influence on the total moment of inertia, $I_{\text{total}}$, of the composite beam. In contrast, in a reinforced concrete beam the moment of inertia due to steel is a small percentage of the total moment of inertia (Fig. 29).

Because concrete can bear only small tension stresses, it must be adequately prestressed in its tension zone. Of the several methods for prestressing, the best known consists of prestressing steel members. Normally, the reinforcing steel is placed in prestressing ducts within the concrete cross section and subsequent grouting of the ducts leads to the composite action of the beam. In theory, it is possible to arrange the steel as a prestressing tieback, but construction, maintenance and corrosion difficulties preclude the use of tiebacks in modern structure.

Typical for composite construction are two methods of prestressing during construction (Lindner, 1980; Rojk Bode and Haensel, 1975): shoring or applying deformations. When shoring is used, auxiliary supports carry the load of the steel girders, when the girders are being erected and while the concrete is being placed (Fig. 30a). In this initial stage, the weight of the concrete is transmitted directly to the ground (Fig. 30b).
After the concrete hardens, the auxiliary supports are removed (Fig. 30c). The load that the supports carried is now transferred to the composite beam such that the girders bears almost all of the dead weight.

If the roadway is constructed of pre-cast concrete members, this method may still be used. In this instance, the shoring occurs after the joints have been filled and the grouting has hardened. Although the processes are elastic, and therefore vary the stresses when carrying permissible loads, under certain conditions, the fluctuating forces do not influence the ultimate loading.

Two steps make up the second method of prestressing composite beams. To get tensile stresses at the top flange and compressive stresses at the bottom one, the girder must be made to curve (Fig. 31a). Either the girder is raised by jacks, or in the workshop it is cambered according to the line of deflection. After the concrete has been placed and has hardened, the girder is released (Fig. 31b). This introduces compressive stresses in the concrete slab and tensile ones in the lower flange of the beam.

For long bridges the jacking and releasing distances are prohibitively great. With a 400m bridge, jacks up to 6m are needed in order to attain a constant bending moment in the inner area of the continuous girder (Fig. 32a).
Thus Roik (1961) devised a method that introduces deformations section by section (Fig. 32b). With the same 400m bridge, the deformations need be only 0.4m. For this method, jacks in the concrete plate produce the needed supplementary angle, by inducing a moment in the deck. In Fig. 33 is an example of the angle prior to application of the moment.

Although prestressing by means of external structures is characteristic for composite girders, the method is not usually used with prestressed concrete. This is because in prestressed concrete, the dead weight is greater and the creep is more noticeable. Thus, only a small part of these states of inherent stress can be used for design purposes.

Cross Sections for Composite Structures

The cross section of a composite bridge is composed of: main girder, cross girder, concrete plate, and mechanical shear connectors. The composite action occurs just in the longitudinal direction of the bridge or in both directions (Fig. 34). Simple T-beams and box girders achieve composite action in the longitudinal direction only (Fig. 34 a, b, c); supplementary cross girders produce the action in both the longitudinal and the cross directions (Fig. 34 d,e). With the cross beams, which can be either plain web girders or trusses, the loads are
transmitted by both the concrete plate and the supplementary girders.

Fig. 35 illustrates box girder which will have a roadway consisting of prefabricated parts prior to erection. Half beams stiffen the lower flange of the box girder, and intermediate cross frames help maintain its shape under torsional loading.

Connectors

Because the steel girder and the concrete plate must act as a homogeneous cross-section, shear forces are transferred in the joint between steel and concrete. For calculating these forces, the elastic theory is used following normal procedures. The theory is only approximate for the failure state, because once the steel enters its plastic range, the distribution of shear forces changes. In Germany, the shear connectors for bridges, must be checked under service loads and in the limit state.

In bridges, shear forces can be transferred by mechanical shear connectors, anchors, or friction, but not by adhesion (Lindner, 1980, Maeda, 1981).

There are several types of mechanical shear connectors (Fig. 37) although the most common is the headed stud (Fig. 36). An array of spirals can increase the load-carrying capacity of the joint. Another type is used in connection with reinforcing bars; yet another is
constructed by four-cornered bars. In many older structures, reinforcing bars were used in conjunction with headed studs. But the high cost of welding has limited the connectors in modern structures to headed studs.

Tests determine the allowable forces of these shear connectors (Fig. 38). Experiments, at the Technical University of Berlin are investigating the load carrying capacity of headed studs with spiral (Roik and Lindner, 1972).

When prefabricated elements are used, the gap in the slab must be filled after the connectors are placed. Although headed studs are again the most common type, friction-grip, bolt are sometimes used in this method of construction (Fig. 39.)

**Ultimate Load-Bearing Capacity**

Because of the relationship between load and stressing for composite girders, the failure state must be investigated. Following the standards used in Germany, it must be proved that the real moments under factored loads are less than the ultimate load-bearing capacity of the cross section.

To calculate the ultimate capacity, the concrete and steel components must be considered separately. Fig. 40 shows the stress-strain diagram for concrete; the dotted line can be used for simplicity.
The ultimate plastic load-bearing capacity is calculated with a simplified procedure. (In comparison with other methods, the errors resulting from this one are very small (Roik, Bode and Haensel, 1975,)) To achieve equilibrium, the pressure force in the area of the effective width of the concrete has to equal the tension force in the steel girder: \( T_s = C_c \). From this equation, \( x \), the distance from the upper edge of the plate to its neutral axis can be calculated. The total plastic moment then becomes the force multiplied by the lever arm:

\[
M_{pl} = T_s \left( z - 0.5 \cdot x \right)
\]

For this calculation, the reinforcement must be taken into account. And if longitudinal forces are present, they must be considered in the statement of equilibrium. The shear forces are absorbed by the web of the steel girder. Their influence can be neglected if \( Q_y \) is less than \( 0.3 \cdot Q_{pl} \) (Fig. 41). If a reduction is necessary, in all cases, the moment acceptable by the flanges is held constant.

State of Service Loads

The composite deck is checked the same way as are steel constructions and concrete structures. The influence of creep and shrinkage must be considered carefully, and the simplest method uses an imaginary modulus of elas-
ticity (Fig. 42) (Lindner, 1980, Roik, Bode, and Haensel, 1975).

With this method, the cross section of the concrete, \( A_C \), is transformed into an imaginary equivalent area of steel, \( A'_C \). The reduction factor for short-term loading is derived from the proportion of the elastic module, the modular ratio, \( n_o = \frac{I_s}{E_C} \).

If the influence of creep is taken into account, the factor \( n_\psi \) is used. It depends on the creep value, \( \psi ' \), and the coefficient of creep, \( \psi \). Differential equations must be solved to determine \( \psi \). These equations are dependent upon the respective cross section and the kind of load on that cross section. To simplify for constant forces, \( \psi \) can be taken as 1.1, which means a modular ratio of approximately 23. With nonconstant moments, the coefficient of creep is taken as 0.52 (a modular ratio of 15). And for release, \( \psi \) is taken as 1.5 yields a modular ratio of 30.

**Conclusion**

I defined the orthotropic steel deck plate and discussed its structural form, commented on ways to calculate plates and girder grillages and showed construction details of existing structures, then gave an overview of the characteristics of composite girders, the different methods of prestressing, the form of the cross sections, and the
design of connectors for transmitting shear forces between steel and concrete. Finally I gave equations for checking the ultimate load-bearing capacity and the capacity under working loads.
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FIGURES

Figure 1: Arrangement of the main components.

Figure 2: Orthotropic steel deck plate under construction in Regensburg, West Germany.

Figure 3: Cantilever arm supported by inclined columns.

Figure 4: Area between the main girders. Shown are the cross girders, the ribs and the space between the roadways.

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Figure 16: Transverse distribution, effect of the number, $n$, of cross girders (HTR = main girder, QTR = cross girder).

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Figure 19: Construction of the orthotropic plate for a railway bridge.

Figure 20: Joint of the ribs.

Figure 21: Joint of a flat plate rib with a weld.

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Figure 23: Joint between a hollow rib and an open rib.

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Figure 27: Lower part of a box girder at the place of previous erection.

Figure 28: Upper part of the main girder at the prefabrication site.

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Figure 30: Shoring for composite beams.

Figure 31: Prestressing by introducing deformations.

Figure 32: Prestressing by introducing deformations section-by-section.

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Figure 38: Example of a procedure for testing spirals and headed studs.

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Figure 42: Method of imaginary modulus of elasticity.
CHAPTER 4

DAMAGE TO BRIDGES AND ITS CAUSES,
MAINTENANCE OF EXISTING BRIDGES

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Karl Blechen's painting "The Construction of the Devil's Bridge" (Fig. 1 shows a section) is based on the following fable:

Once, in the Swiss canton of Uri, the people wanted to build a bridge but did not know how. The devil was asked for assistance, and he assented after receiving a promise that the soul of the first person to cross the bridge would belong to him. Two days later the bridge stood completed, and the Swiss chased an old billy goat over it. The devil felt cheated. From that time since, he has plotted his revenge: the destruction of all bridges.

The fable has two morals:

1. From olden times, bridge builders have fought against the destructive forces of the devil. The battle begins the moment the structure is completed.

2. Ever since, civil engineers have left the dedication of a new bridge up to a politician.

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MAINTENANCE OF LARGE, OLD BRIDGES

The long-term influence that a harmonious environment has on the spiritual development and the well-being of an individual is well known. Just as we are rediscovering today the aesthetic value of buildings from other eras, we are likewise striving to restore venerable bridges (Fig. 2). Repairing damage, strengthening, or widening these structures should not be allowed to affect the original beauty. The technology of bridge construction, such as mobile scaffolding, pre-stressing methods, injection techniques and surface protection, can be adapted to maintenance of other bridges.

Unfortunately, these lessons - as illustrated in Fig. 3, are exceptions. The overwhelming majority of our bridges which are no more than 30 years old, require for maintenance a yearly expenditure of 10 to 20 DM/m ($5-10/m) of bridge surface area or approximately 1% of the overall construction costs. In view of the almost 17 million square meters of total bridge surface in Germany alone, this is a considerable sum. Already there are voices clamoring for the establishment of a new disciplinary field for the civil engineer: maintenance.

Ultimate collapse (Fig. 4) or cases of construction catastrophe are not addressed herein. Luckily, most
extensive damages are less common. Collapse is generally rare, and in most cases occurs at the temporary supports and construction footings (Fig. 5). The next most frequent collapse is due to over-stressing from natural forces such as earthquakes or floods (Fig. 6).

But three-quarters (71% according to Blunk) of all instances of damage are caused by corrosion of the reinforcement. Stress-crack corrosion and the hydrogen brittleness of prestressing steels no longer occur. Today's reinforcing steels are not susceptible to these effects, and in 1962 calcium aluminate cement was forbidden for use in building elements. Corrosion due to friction plays a minor role in pre-stressed structures with a bond, but may occur in pre-stressed structures without a bond - a subject of current research (Patzak, 1978). Along with corrosion owing to chloride, surface-destroying oxygen corrosion remains an ever-present hazard - as old as concrete construction itself (Figs. 7 and 8).

**Concrete Covers and Impervious Joints**

The German National Standards (DIN) for the minimum cover layer of concrete are favorable to concrete construction technology. A necessary least dimension for practical tolerances is not specified. Thus the lowest values automatically become average values. Dimensions of 0.5 to 1.5cm could be suggested as tolerances. In addition,
the very important factor in concrete mix, the water-cement ratio, is also unspecified in the standards. Research results, which offer some correlation between the parameters such as the thickness of the concrete deck, the crack widths, and the environmental conditions, and the corrosion patterns are incomplete and therefore, at present, of limited value.

To illustrate (compare to Fig. 9): During hydration of the cement, only a portion of the mixing water is chemically bound and integrated into the hardened material. For a complete hydration the proportion of water needed is approximately 25% by weight of Portland cement. If, in addition, one considers the water that does not evaporate until the temperature reaches 100°C, then a water-cement (W/C) ratio of 20% would be chemically sufficient. Every further addition of water that increases the W/C ratio necessarily requires a corresponding pore spacing, or else upon evaporation capillaries are formed. The gel resulting from the hydration process with a W/C ratio of 0.40 is just sufficient to close these capillaries. Additional water inevitably leads to porosity. The critical limit for imperviousness lies at 25% of the volume of the hardened cement paste. This limit is reached during complete hydration of freshly poured concrete having a 0.60 W/C ratio and during partial hydration of 70% in concrete
having a W/C ratio of 0.45. The highest allowable value according to DIN 1045 is 0.75, which is too great. In the future it may be necessary to limit this value to 0.60.

Essentially porosity has the following effects: excessive reduction of strength, accelerated corrosion of the reinforcing steel by addition of oxygen, accelerated carbonization of concrete by carbon dioxide and loss of the alkaline protective effects for the reinforcement, and admission of corrosive substances such as chloride.

These are the main reasons that a 2cm-thick, strong, concrete cover may prove adequate (W/C = 0.45) in one case, and may lead to rusting in another after only a few years (W/C = 0.7). Although a practical correlation is unknown, the following serves as a rule of thumb: An increase in the W/C factor of 0.1 requires an additional 0.5cm of concrete decking.

Along with the concrete deck, the following factors are essential for good protection against corrosion:

- The concrete should be as dense as possible, especially in the external zones. A stable formwork and vibration zones must be achieved near the edges as shown in Fig. 10, and excessive concentration of the reinforcement must be avoided.
- The W/C ratio, adjusted to achieve the necessary workability, should be as small as possible. Liberal cross-sectional dimensions are, as a rule, a precondition of durability. Thus, striving for ever-more filigreed cross-sections has its limits.

- Only cement that has changed into Ca (OH) creates alkaline protection, a process requiring a high curing rate. Only intensive post-treatment (water treatment) leads to a high curing rate in the layer of concrete decking. But it must be mentioned here that surfaces dry out too quickly and the actual rates of hydration at times do not exceed 70%. A few German construction authorities therefore require, along with intensive post-treatment, a minimum amount of cement greater than DIN 1045.

**Influence of Melting Salts**

In the harsh winter of 1978/79 alone, 13 ton/km of melting salts in the form of NaCl were spread on Germany's interregional highways and 43 ton/km (the greatest amount to date) were used on the autobahns. In other winters, large quantities of melting salts has also been used.
Over several years this practice results in damage, not only at points of direct contact but also on all structural members (Fig. 11). Especially imperiled are those places with a high W/C ratio and incomplete protection, such as blocking plugs, anchorages near the surface, working joints, and also places beneath previous courses of concrete and permeable expansion joints.

The physical effects of melting salts on concrete are now mainly seen as the result of temperature shocks due to endothermic chemical reactions. Chemical decomposition of the hardened cement paste may also play a role, but conclusive evidence is lacking. On the other hand, the most immediate effect on the reinforcing steel is clearly that described as pitting corrosion, which is especially dangerous, because as soon as a small amount of chloride reaches the steel, the ion attacks the metal locally and can lead to sudden collapse of the structure.

Avoiding NaCl as a melting salt is the simplest solution. Although there have been recent attempts to reduce its use drastically, complete avoidance appears impossible because there is still no equally effective substitute that leaves bridges ice-free in winter. Thus the following measures are the mainstay of defense:
Concrete parts should be fabricated with the greatest possible density, and in special cases, with a 5% component of air-entrained voids.

The superstructure should be coated with epoxides, especially in areas where the thruways are connected to it (Fig. 12).

Endangered cables and pre-stressing tendons must be easily exchangeable and capable of being post-tensioned. Thus the French write about empty tubes for introducing cross-sectional stressing tendons during post-tensioning. Exchangeability of the cables in [cable-stayed] bridges is a prerequisite of construction.

The bridge must be inspected continuously. Fig. 13 shows how the inside of a caisson can look.

And if the damage is already present, then repairs must be made as quickly as possible, preferably by torquetting.

An extensive practical experiment on German bridges (Harig, 1980) has up until now found that imperviousness to melting salts does not depend on
the quality of the concrete, the type of cement and the additives, but rather on good manufacturing technology and intensive post-treatment.

**Damages Due to Design Specifications**

As design errors have occurred in the past, they have been quickly detected and eliminated. There have been five main errors in design.

1. The level of complete prestressing has been miscalculated when concrete was pre-stressed. Because no calculated tensile stresses were available, it was believed that the only solution was very weak, loose reinforcement. There was even a misconception that pre-stressed concrete was an entirely new construction material. Today students are taught that prestressed concrete is still reinforced concrete with an artificially applied load; only strong and well-distributed, loose reinforcement can keep the inevitable cracks so fine that they do not affect durability; and the main stresses are caused by temperature. A structure should not counteract stresses with all its might but should be able to yield to them without sustaining damage. A high degree of prestressing makes the structure brittle.
Encouraging reports from Switzerland on bridges with only segmental prestressing substantiate this hypothesis. However, uncertainty still surrounds control of the crack problem, although qualitative correlations have been established and initial structural recommendations have been obtained (Falkner 1969, Leonhardt 1977, Noakowski 1980). In the long term, this remains a research problem of the highest order.

A case in point is the following situation. According to conventional wisdom, many thin, closely-spaced bars lead to a more even distribution of cracks than do fewer, thicker bars, but with respect to a structure's durability, one might well ask whether a few, thick reinforcing bars with an already-acceptable crack width (0.2mm) are better than several thin bars with fine cracks (0.1mm). An optimum combination of thin outer bars and thicker inside bars seem to be most promising. Experiments in Stuttgart substantiated this hunch. Such structures also appear to be more robust and more emenable to rehabilitation.

This idea is not new. It goes back to the "Benzinger Mesh," known since 1934 (Fig. 14), which possessed smooth reinforcing bars for the then-allowable prestressing values that had been permitted by experiments, that today apply first to high-value reinforcing steel (240 KN/cm2).
(2) The average values of these dimensions under indirect arrangement of a bridge girder originally were underestimated. Especially disregarded was the high slope of the bearing reaction by means of reinforcement in the area between main and cross girders (Fig. 16). Only consistent, strict attention, to the skeletal model of MORSCH with braces inclined at 45 leads to over-dimensioning (Fig. 17). Damaged bridges were rehabilitated with vertical tensioning members without bond that were added on later (Fig. 18).

(3) Curved tensioning tendons produce deflection stresses. With inclined, curved planes, their horizontal components were easily overlooked. In conjunction with the old philosophy of prestressed concrete, these cross-sections were given much too little cross-reinforcement and consequently split in wide, longitudinal cracks (Fig. 19). Extremely ugly outer tensioning members eliminated the initial danger. Attaching steel butt straps (Fig. 20) would be a the newer method of repair.

(4) Structures built in segments (Fig. 21) necessarily have joints. The ends of the segments once were prime spots for anchoring tensioning cables. Although the cables were lengthened by coupling, and the subsequent section was tensioned against the segment just placed, thus compressing the joint, this method produced special sectional forces,
which initially were overlooked. The conceptual model [5] shown in Fig. 22 clearly illustrates the resulting stress and strain. Imagine a gap between two segments in the region of the tensioning members, but joined through pressure over a metal pipe. Now the prestressing deforms the cross sections, such that the anchoring force of the first is diffused completely and that of the second segment is concentrated. The special dimensions of the coupling now must make both cantilevers compatible. These can be thought of as single, concentrated forces, so that their area of initial force has to be further reinforced. If the joint site is now filled with too little reinforcing material, cracking inevitably results (condition II), with significantly greater stress and strain in the tensioning members (Fig. 23).

These coupling areas lie mainly in places with zero moment. Because the calculated position of zero moment is different from the actual position, a safety factor is ineffective; zero multiplied by anything is still zero. Of most help are structural rules or flat-rate and/or stiffness-related additions to the design dimensions of sections.

Damaged bridges are rehabilitated by introducing additional longitudinal tensioning members without bond or by fastening steel butt straps to the interior of the caisson.
(5) The first four problems occur sporadically. More difficult to diagnose, but occurring constantly, is the danger that bridges are designed by engineers who have too little experience and an insufficient breadth of knowledge, and that errors attributed to construction are already committed on the drawing board. Here again, the main mistakes are inadequate concrete cross sections and reinforcement that is too dense.

(6) Joints in bridges constructed in spans are avoided if, at every one of these spots, an open joint is provided with an expansion joint. This method has been tried in connection with specially-developed expansion joints, but has not proved satisfactory.

Today, the longest possible jointless superstructures are planned with correspondingly heavy transition elements. Scrimping here would lead to self-deception. The frequency of replacements, their costs and the effects of traffic eat up any conceivable savings many times over.

CONSTRUCTION PROBLEMS

Our structures are actually much too complicated. As the qualifications of the skilled worker and of the accompanying supervisors at the construction site decrease, it is ever more difficult to ensure a guarantee of "work well done".
The average age of all German construction foremen is over 50. Researchers and the development divisions of firms must produce structures simpler in every possible way, without increasing costs and while also permitting only the smallest possible aesthetic losses.

However, daily, as in the past, state-of-the-art concrete bridges are being erected (Specht, 1981). An example of just such an outstanding achievement is shown in Fig. 24.

Of special concern were the injected or incompletely-injected tension members with post-tensioned connections. In older, pre-stressed concrete bridges, uninjected metal shafts are found again and again. In the meantime, the discharge cross sections have thus become enlarged, and the main innovation has been the mechanization of both the mixing of the injection mortar and the injecting itself (Fig. 25). Despite undeniable advances, there still remains a minor difficulty that we would only eliminate if, by means of a non-destructive, easily usable testing method, the seam-less joint and the corrosion protection could be verified after construction.

Gammography (the radiographic inspection of concrete sections) requires specialized knowledge and experience, and is only employable in exceptional cases as a research method (Fig. 26). Uninjected tensioning members betray themselves mainly through longitudinal cracks in the girders.
The remaining typical design errors - such as a W/C ratio that is too high, insufficient compaction, and a formwork that is too flexible with resulting corrosion of the reinforcement - have been discussed earlier. The measure that is most often neglected is intensive post-treatment. Because swift construction progress has become the number one economic goal, post-treatment of the concrete has often been minimized.

BRIDGE INSPECTION

Every bridge in Germany must be observed continuously through inspection trips and simple scrutiny - it must be inspected on foot once a year, given a simple check-over every three years, and thoroughly inspected every six years, according to guidelines found in the so-called "Construction Book," which contains all important technical numbers and details.

For this purpose, the highway authorities have created their own inspection teams with specially-trained specialists and modern resources. This concept is based on the realization that only scheduled, regular maintenance, beginning as soon as the bridge is opened to traffic, ensures durability and minimizes total costs, and that only systematic inspection can protect the public from catastrophic failures. Steel-reinforced concrete is a good-natured building material, and as a rule visibly shows
its existing weak points long before they become dangerous. Figs. 27 and 28 show a modern, heavy bridge inspection apparatus with a boom basket for two people; Figs. 29 and 30 depict an apparatus with an observation platform.

A further precaution against excessive stress and strain is the posting of standardized signs at every bridge. Before crossing any bridge, the user finds a sign telling him the calculated traffic load according to the type of bridge, as seen in Fig. 31, for example. The sign indicates that vehicles with a gross weight of up to 60 tons (600KN) can traverse the bridge safely without special permission.

FINAL COMMENTS

The future of concrete bridge construction lies not in setting new records for span length. More appropriate are:

- building structures of high aesthetic value and with well-executed details,

- further rationalizing highly-developed construction methods, without losing reliability or flexibility,
- devising simple and less sensitive structures, as well as developing robust, pre-fabricated materials that are more easily and more reliably manufactured,

- improving quality control methods and extending them to the finished product, by means of non-destructive testing procedures,

- designing structures favorable to inspection and repair, as well as developing long-lasting and cost-efficient repair methods,

- training highly qualified on-site personnel, and designing engineers with solid practical experience,

- reducing administrative restrictions (codes) on responsible engineers (a special problem for Germany), along with providing better information on the behavior of bridges.

- introducing partial pre-stressing and segmental construction with the ultimate aim of making bridge construction less expensive and also increasing the durability of bridges,
improving the quality of structures in every respect. It must be guaranteed that quality has economic advantages for those who make it possible and for those who attain it,

supporting research that puts these efforts on a firmer foundation by increasing and confirming our knowledge; particular attention must be paid to the crack and corrosion problems.

The apparently high frequency of reported cases of damage to our bridges should not leave the impression that Germany is littered with the ruins of her bridges; the opposite is true. As in the past, German bridge construction enjoys high international esteem. Of 1000 bridges, about 20 are damaged and only two of these severely. I have described these bridges in order to demonstrate that one of the major tasks of future civil engineers is creative maintenance of existing concrete bridges, which were constructed with creative enthusiasm.
REFERENCES


FIGURES

Fig. 1 Karl Blechen: "Constructing the Devil's Bridge" (section) Neuespinakothek, Munich.

Fig. 2 An old viaduct that should be maintained.

Fig. 3 Restoration and widening of a bridge on the Nuremberg - Munich highway.

Fig. 4 Collapsed bridge under construction.

Fig. 5 Collapsed scaffolding of a concrete bridge.

Fig. 6 Bridge destroyed by a flood.

Fig. 7 Corrosion of the reinforcement in a longitudinal girder of a bridge.

Fig. 8 Corrosion of the reinforcement of a deck bridge.

Fig. 9 Graph of the hardening of cement with different water-cement ratios.

Fig. 10 Arrangement of vibration paths in
heavily-reinforced cross-sections.

Fig. 11  Bridge facade damaged by melting salts.

Fig. 12  Bridge coated with epoxides.

Fig. 13  A salt pool that seeped into the interior of a box cross-section.

Fig. 14  The "Benzinger Mesh".

Fig. 15  Indirectly-loaded main bridge girder.

Fig. 16  Framework model for calculating the stresses under indirect loading.

Fig. 17  Transversely-rehabilitated bridge.

Fig. 18  Spatially-curved span members in the longitudinal direction of the bridge.

Fig. 19  Rehabilitation through outer cross-girders. Definite forces from pre-stressing cracked the bridge longitudinally.

Fig. 20  Construction with a carriage, requiring a working joint in every span.
Fig. 21 Theoretical model for designing a joint with coupling between tendons.

Fig. 22 Large cracks occurring at joints with tendon coupling.

Fig. 23 The Neckarsheim arch bridge in Germany.

Fig. 24 Mixing and injection machines for tendons with an added joint.

Fig. 25 Radiogrammed concrete member. The light contours show the member, the anchorage and the coupling bell (from left to right). The member extending to the right is torn from the bell.

Fig. 26 Heavy bridge inspection equipment.

Fig. 27 Boom basket for two people.

Fig. 28 Observation platform.

Fig. 29 Detail of a bridge inspection.
Fig. 30 Example of sign indicating the maximum traffic load of a bridge.


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